651

VOLUME 81 SEPARATE No. 651

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

MARCH, 1955

HIGH STRENGTH STEEL BOLTS IN STRUCTURAL PRACTICE

by Mace H. Bell, A.M. ASCE

STRUCTURAL DIVISION

{Discussions open until July 1, 1955}

Copyright 1955 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

PRICE $0

THIS PAPER

--represents an effort by the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final date on which a discussion should reach the Manager of Technical Publications appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings — Separate No. 290). For free copies of this Separate—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

$0.50 PER COPY

HIGH STRENGTH STEEL BOLTS IN STRUCTURAL PRACTICE

Mace H. Bell,[1] A.M. ASCE

Bolts have been used in structural steel connections from the very early days of steel fabrication, and various specifications governing the design, fabrication, and erection of steel have recognized their use under certain specified conditions. Until a few years ago, the types of bolted joints permitted, could be broadly classified as:

1) The "unfinished," "common," or "machine bolt" joint, with 1/16 inch clearance in holes.
2) The "turned bolt" joint with a "close" fit, usually 1/50 inch clearance in reamed or drilled holes.
3) The "ribbed" or "rivet bolt" joint, where the "fluted" shank of the bolt is slightly larger than the holes in the connected material and bearing between the bolt shank and the plies of connected material is obtained by a "driving fit" at the time of installation.

Approximately seven years ago an entirely new concept of bolted connections for structural connections came into being as a result of the research work and investigations sponsored by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation. The principle of this connection places it in a different category from that of the other types of bolted connections already mentioned. This joint depends upon friction between the faying surfaces of the connected members for stress transfer. To round out the list of broad classifications of bolted connections, it can be described as:

4) The pre-tensioned "high strength steel bolt" joint, with hardened washers and 1/16 inch clearance in holes.

Since January 15, 1947, when the Research Council was organized, this new type of joint has had a growth in acceptance and usage that has seldom been matched by any other newly introduced method in structural steel assembly. It has been accepted by both structural and bridge engineers as a connection that is suitable not only for static loads, but also for all ranges of dynamic loads.

Prof. Munse has told you in Proceedings—Separate 650 of the laboratory and field research work that led up to the development of the Specifications for Assembly of Structural Joints Using High Strength Steel Bolts, first issued in January 1951, and revised early this year, the current effective date now being February 1954. The discussion that follows is confined to the practical aspects of use of this new "high-strength steel bolt" in structural connections.

Within a very short time after the Council's Specifications were first issued in 1951, the American Institute of Steel Construction endorsed them.

1. Assistant to Director of Eng., Am. Inst. of Steel Construction, Inc., New York, N. Y.

This action constitutes full acceptance of this method of connecting structural members by the Institute. Therefore, when the A.I.S.C. Specification is next revised, these provisions controlling the use of High Strength Bolts will be incorporated into the body of that document. Meanwhile, the two specifications are referred to jointly in any job specification in which it is intended that high strength bolting be used.

In a similar manner, the American Railway Engineering Association, and the American Association of State Highway Officials, have accepted the Research Council's Specifications with only minor editorial changes, thus permitting the use of high tensile bolted connections in railway bridges, and in highway bridges.

From the practical viewpoint, there are six principal features of the Council's very brief specification that should be emphasized and discussed in some detail.

The first important feature is that the connections shall be designed and fabricated to conform to the requirements of applicable existing codes or specifications for riveted structures. A high-strength bolt of the same diameter may be substituted for each rivet of ASTM A141 quality in any joint so designed.

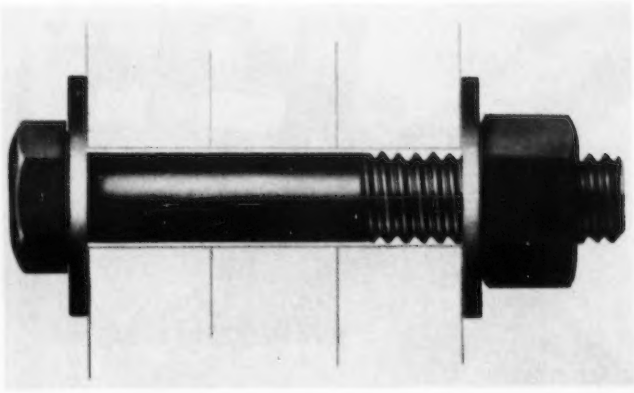
The cut-away view of Figure 1 pictures a high tensile bolt in a three ply structural connection. The joint is made up with a bolt, a heavy nut, and two hardened washers, assembled into standard 1/16 inch clearance holes as used for rivets. There are certain differences in physical appearance that distinguish it from the "ordinary" bolted joint used in structural practice. These are indicated in the lower sketch of the figure. They are: First, the use of heavy hardened washers under both the bolt head and the nut; Second, the use of heavy semi-finished hexagon nuts; Third, a distinctive "bolt head marking" of three radial lines, as required by ASTM Specification A325; and Fourth, the bolt is black in color from the heat treatment given it.

A point of special interest to engineers, that might be mentioned here, is that no special provision for locking the nut is required. When the bolts are properly installed, and the nut is run up to induce the high tension specified, both field and laboratory tests have proven that there is no tendency for the bolt to lose tension even under millions of cycles of loading. The friction of the threads, and of the bearing surface of the nut, keeps it permanently tight.

This photograph, Figure 2, shows the detail of a truss to column connection. From the appearance of the joint, it is just like a riveted connection, except that high-strength bolts and washers are used in the field connection. The workman at left is tightening the bolts with a pneumatic impact wrench, while his partner holds a wrench on the bolt to prevent its turning in the early stages of tightening.

The second important feature of the Council's specification covers the bolts, nuts, and washers. These are all required to conform to ASTM specification A325, and to the dimensions established by the American Standards Association. The bolt is made from a medium carbon steel, and is heat treated, quenched and tempered. The nut is made from a similar steel, but no heat treatment is required. The hardened washers are either quenched and tempered, or carburized, quenched and tempered.

Figure 3 illustrates the typical stress-strain curve obtained in tests of A325 bolts. The line is straight up to the elastic limit, then curves gradually without any clearly defined yield point to an ultimate strength that is some 70% above the recommended pre-load to which the bolts are tightened. Note that the strain above the yield point is 175 greater than the total elastic strain.



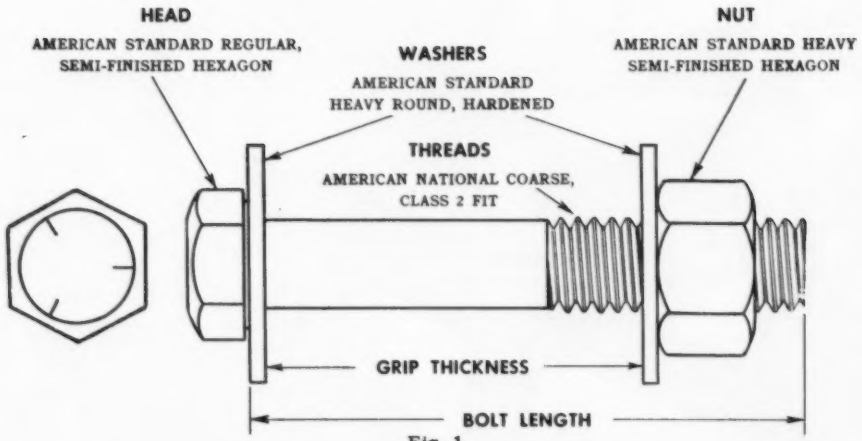
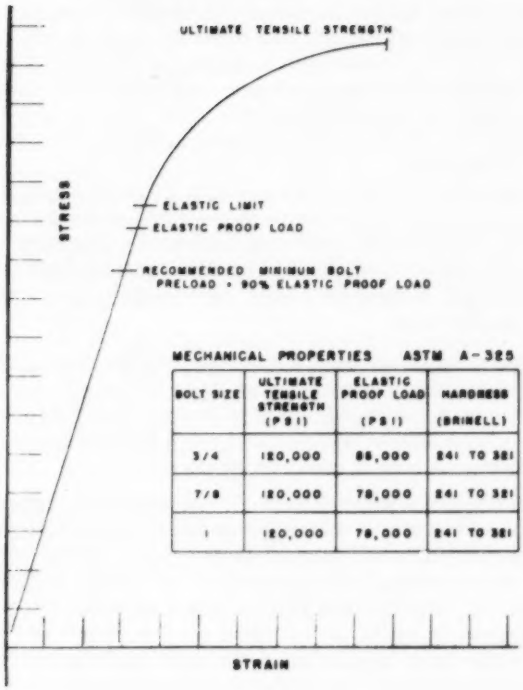

HEAD
AMERICAN STANDARD REGULAR, SEMI-FINISHED HEXAGON
WASHERS
AMERICAN STANDARD HEAVY ROUND, HARDENED
NUT
AMERICAN STANDARD HEAVY SEMI-FINISHED HEXAGON
THREADS
AMERICAN NATIONAL COARSE, CLASS 2 FIT
GRIP THICKNESS
BOLT LENGTH

Fig. 1

Fig. 2



MECHANICAL PROPERTIES ASTM A-325

BOLT SIZE	ULTIMATE TENSILE STRENGTH (PSI)	ELASTIC PROOF LOAD (PSI)	HARDNESS (BRINELL)
3/4	120,000	85,000	241 TO 321
7/8	120,000	78,000	241 TO 321
1	120,000	78,000	241 TO 321

Fig. 3

The mechanical properties required for three commonly used bolt sizes are tabulated to the right of the curve, and the relative position of the elastic proof load, the elastic limit, and the ultimate strength, as well as the recommended minimum bolt pre-load, are all indicated on the curve. The tension test, up to the elastic proof load, is determined by axial loading, as indicated in the sketch of a bolt head to the left. The ultimate load is determined with a 10^{o} wedge under the bolt head, as shown in the sketch to the right.

Two conclusions can be drawn from this chart. First, it is evident that there is no danger in tightening the bolts to tensions considerably above the required minimums, called for by the Council's specification, since there is a considerable reserve strength in the bolt beyond that point. Note that the strain in the bolt at its ultimate strength is approximately 3 1/4 times the strain at the recommended minimum bolt pre-load point on the curve. And, second, the bolt strength, at its proof load and upper limits, is not materially affected when the bolt head and nut bear on non-parallel surfaces up to a slope of 1 to 6, or 10 degrees.

The third important provision of the Council's Assembly specification requires that the contact surfaces, including those adjacent to the washers, shall be descaled, or carry the normal tight mill scale, and they must be free of dirt, oil, loose scale, burrs, pits, and other defects that would prevent solid seating of the parts. It then sets forth two requirements regarding shop painting which are quoted here:

> "Contact surfaces of joints where slippage into bearing is acceptable may carry a shop coat of protective paint or lacquer."
> "Contact surfaces of joints subjected to stress reversal, impact, or vibration, or where stress redistribution due to joint slippage would be undesirable, shall be free of paint or lacquer."

Most bridge specifications and the A.I.S.C. Specification ordinarily permit shop paint in the connections, and the engineer must therefore give consideration to the requirements of the various joints in a specific job before arriving at a decision as to which provision is applicable. From the standpoint of strength of a connection, subject to static or pulsating loads, the Council has made shear tests where the bolts have slipped into bearing, and found the shearing strength of the joint considerably stronger than that of riveted joints. From the standpoint of economy, the omission of paint, with the additional work of indicating such areas in the drafting room, and masking or marking in the shop before painting, is an expensive operation reflected in bid prices.

From the standpoint of accepting some slippage, which could probably not exceed 1/16 inch even with perfectly matched holes, the engineer should consider; first, the direction in which the slip can occur and the consequence of maximum slip; and second, the rate of slip under service conditions. The laboratory investigations that have been performed so far on painted surfaces have indicated a long time slip and if the joint is loaded to its full design capacity with static loads, we can expect slip. On the other hand, with normal loads, which are apt to be less than maximum, it may take hours or days before the bolts slip into bearing. It must be remembered that for the usual condition, a joint load is made up of a combination of dead and live load, and that when we have live loads of a transient nature, such as wind loads, and certain classes of dynamic loads, the joints are stressed to full capacity only for short periods of time.

Now, if we apply these two considerations to the connections of a tier building for example, we can reason as follows: The milled column splices

cannot slip as they are already in bearing. The web or seat connections could move downward only, and should this occur, the effect would be negligible. The wind connections are subject to fluctuating short time loads which normally would not cause slip. It is very doubtful whether painted joints would experience any greater slip under such conditions, than that which has occurred in riveted joints.

The fourth important feature of the Council's specification provides that flat washers may be used if the abutment surfaces adjacent to the bolt head and nut do not have a slope of more than 1 to 20 with respect to a plane normal to the bolt axis. There are two slopes on structural shapes rolled in this country. These are given in the AISC Manual of Steel Construction, and can be summarized as follows:

	Slope	Percent	Degrees
Certain WF Shapes	1 to 20	5	2° - 52'
American Std. Beams and Channels	1 to 6	16 2/3	9° - 28'

This clause of the specification thus permits the use of flat washers under bolt heads when bolts pass through the flanges of WF shapes, but requires the use of beveled washers for flange connections for American Standard beams and channels.

This photograph Figure 4, shows a 36 WF floor-beam to plate girder connection in a single track trough-type railway bridge. Note that the lateral bracing gusset plate at the bottom of this floor-beam, one with a 5% slope, is bolted with square beveled washers, as required by the 1954 specification. Figure 5, pictures the connection of a crane runway girder to a column, and shows a splice in the girder to the right. Note that flat circular washers are used in the angle to web knee brace connections, in the girder web splice, and on the outside face of the girder flange splice. The square beveled washers seen on the inside face of the girders upper flange, were installed under the earlier 1951 specification. In both cases, the beveled washers could be omitted under the current 1954 specifications, had the bolts been reversed.

The fifth important requirement of the Council's specification is that all bolts must be tightened to induce a required minimum bolt tension equal to at least 90% of the specified minimum proof load of the bolt.

Figure 6 is reproduced directly from the specifications to show these requirements. It should be noted that the tension values shown are total tension per bolt, and not psi. The first column shows bolt size. The second column gives a recommended bolt tension for calibrating wrenches. This is not a requirement but is recommended as a "target" value which will insure that all bolts will be tightened to the required minimum tensions shown in the third column. If wrenches are calibrated to produce a given average tension, the actual tensions in a group of bolts making up a connection will vary from that average by perhaps 5% to 10%. For this reason the "target" value is set at 15% above the required minimum tension. The fourth column gives the approximate equivalent torques needed to produce the specified minimum tensions. These, too, are not a requirement of the specification. They are experimental approximations obtained from studies of torque applied to non-lubricated bolts and nuts. Such variables as the amount of lubricant on threads, and the condition of the thread, nut and washer surfaces, all affect these values considerably. They are included as a guide which is useful in the first steps of calibration to determine the requirements of a particular job application.

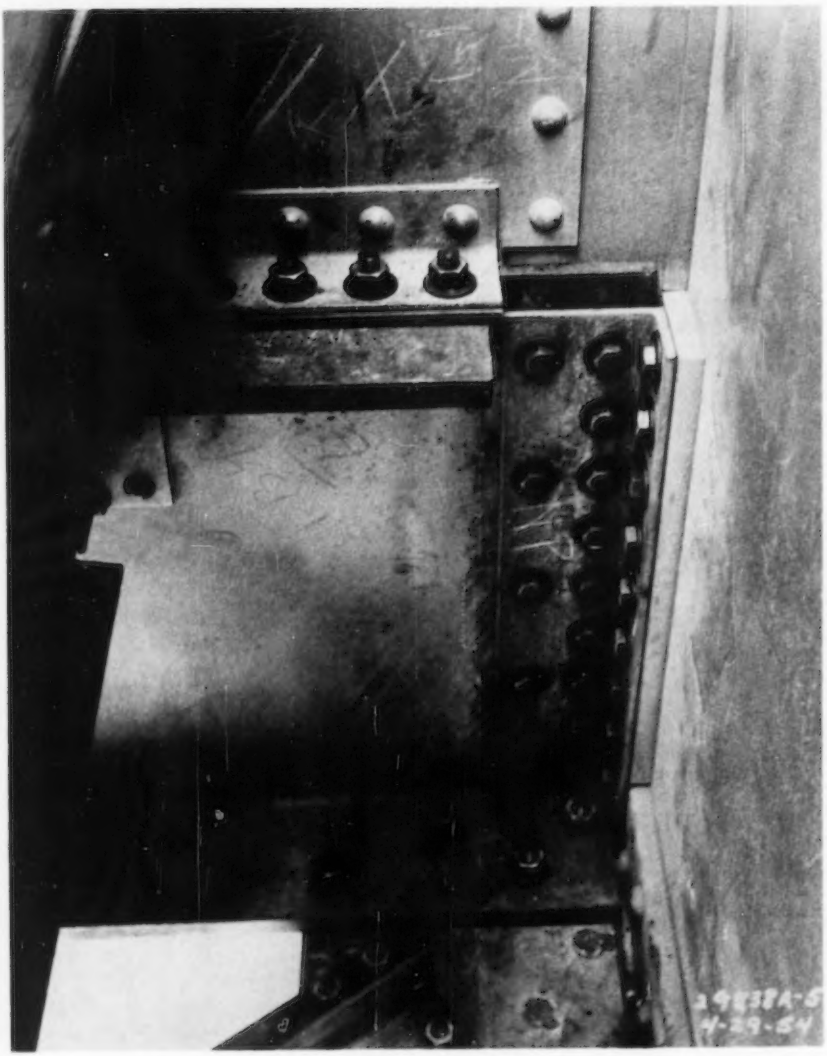


Fig. 4

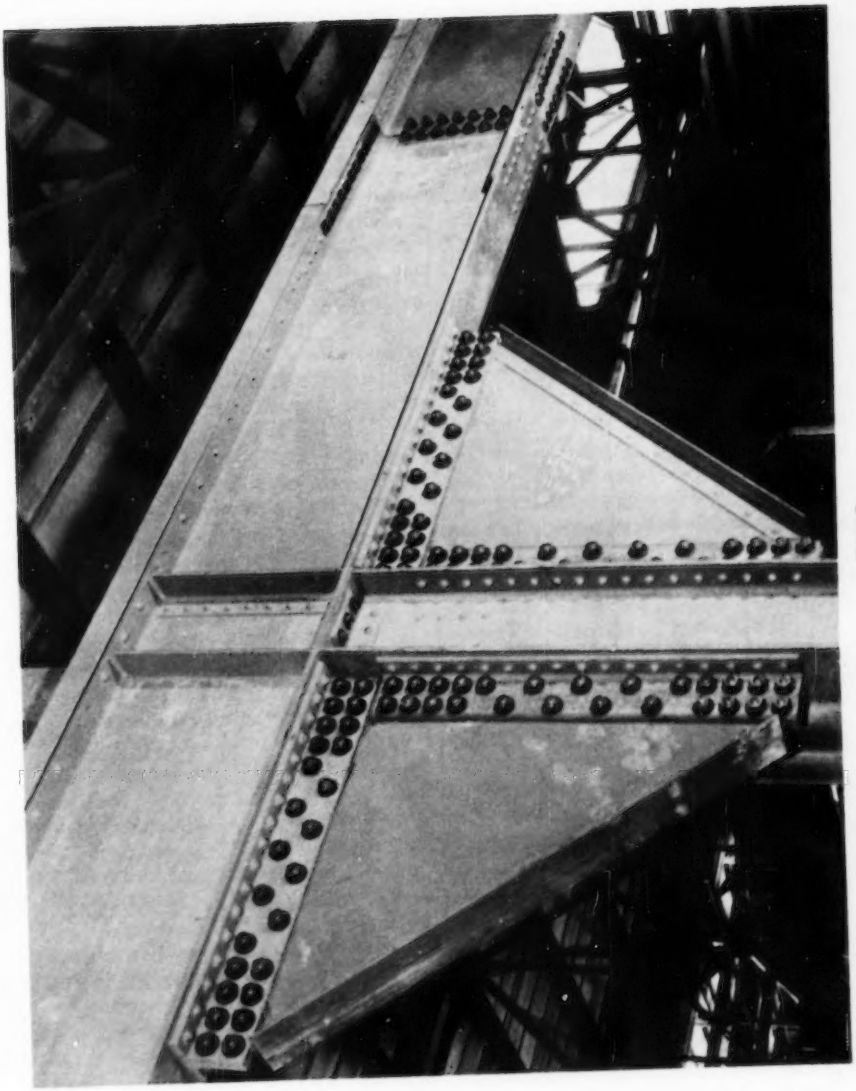


Fig. 5

Bolt Size (in.)	Recommended* Bolt Tension for Calibrating Wrenches (lb.)	Required Minimum** Bolt Tension (lb.)	Approximate*** Equivalent Torque for Required Minimum Bolt Tension (lb. ft.)
1/2	12,500	10,850	90
5/8	20,000	17,250	180
3/4	29,000	25,600	320
7/8	37,000	32,400	470
1	49,000	42,500	710
1 1/8	58,000	50,800	960
1 1/4	74,000	64,500	1350

* Approximately 15 percent in excess of the Required Minimum Bolt Tension.
** Equal to 90 percent of the minimum Proof Load of Bolt (ASTM A325). There is no recommended maximum bolt tension.
*** Equal to 0.0167 lb. ft. per in. bolt diameter per lb. tension for non-lubricated bolts and nuts. Values given are experimental approximations. If torque rather than tension is to be measured, the torque-tension ratio shall be determined by the actual conditions of the application.

Fig. 6

Although manual wrenching of the bolts has been used where only a few connections are required, it is slow, cumbersome, and relatively expensive for general applications. The size of wrench required for tightening the nuts on 7/8 inch bolts can be judged by this photograph, Figure 7. The workman at the right is using a hand torque wrench. For these particular bolts, he must exert a pull of approximately 100 lbs. As contrasted to this, the use of an electric or pneumatic impact wrench is speedy, and requires a minimum of effort, Figure 8.

The usual method of assembling joints is to use the pneumatic impact wrench. In addition to having the advantages of speed and ease with which high torques can be applied, the pneumatic impact wrench lends itself readily to use of a remarkably dependable method of reproducing uniform tension in the bolts. Field experience on many different jobs has proven that by controlling the air pressure fed to the wrenches, they can be calibrated to deliver the required tension, without any reference to torque.

Several different devices have been developed which are used in checking the tension induced in the bolts by impact wrenching. By inserting a pressure regulator in the air supply line to each wrench, the wrench can be calibrated, or adjusted to "stall" when the desired tension is reached. In practice a number of bolts of the size being used are tested in the device, and the air pressure and approximate time of operation required to actually "stall" the wrench are determined. At the point of "stall" the wrench continues to impact, but there is no visible turning of the nut. This can be determined by watching a series of crayon marks on the wrench chuck.

One such device is shown in Figure 9. It consists of a 60 ton hydraulic puller mounted on a steel base. By means of a yoke arrangement at the base of the puller, the tension induced in the bolt by the impact wrench is read directly on the gauge mounted on the jack. In the photograph, the bolt being tested is hidden by the supporting frame. The nut bears against the frame, and the bolt head bears against the yoke which is in turn connected to the heavy threaded rod extending through the jack.

Two similar devices using hydraulic jacks and gauges to read the induced tension direct are shown in Figures 10 and 11.

All of these devices are rugged, simple to operate, and easy to move around on the job site. They can be used to calibrate a wrench in just a few minutes. They can also be used to train wrench operators, and to re-check wrench adjustments, and operator performance at any time.

Other devices have been used for calibration. One of these consists of a heavy steel testing yoke made up of two plates separated by two spacer bars at opposite edges. The deflection of these yokes under increments of load is first measured in a hydraulic press, and a calibration table is prepared. When a bolt is placed in the yoke and tightened, its tension can be determined by measuring the yoke deflection with micrometer callipers. Another device consists of a steel tube with electrical strain gages and a meter calibrated to read the bolt tension directly in pounds, Figure 12.

Another method of reproducing uniform tension in bolts, which is currently under investigation in the Association of American Railroads laboratory, gives promise of an even simpler operation than those described. It is based on the fact that once the connected plies are seated, there is a definite relationship between turns of the nut and the tension that is induced in the bolt. The method will be reported in a paper to be presented to the ASCE in the near future.

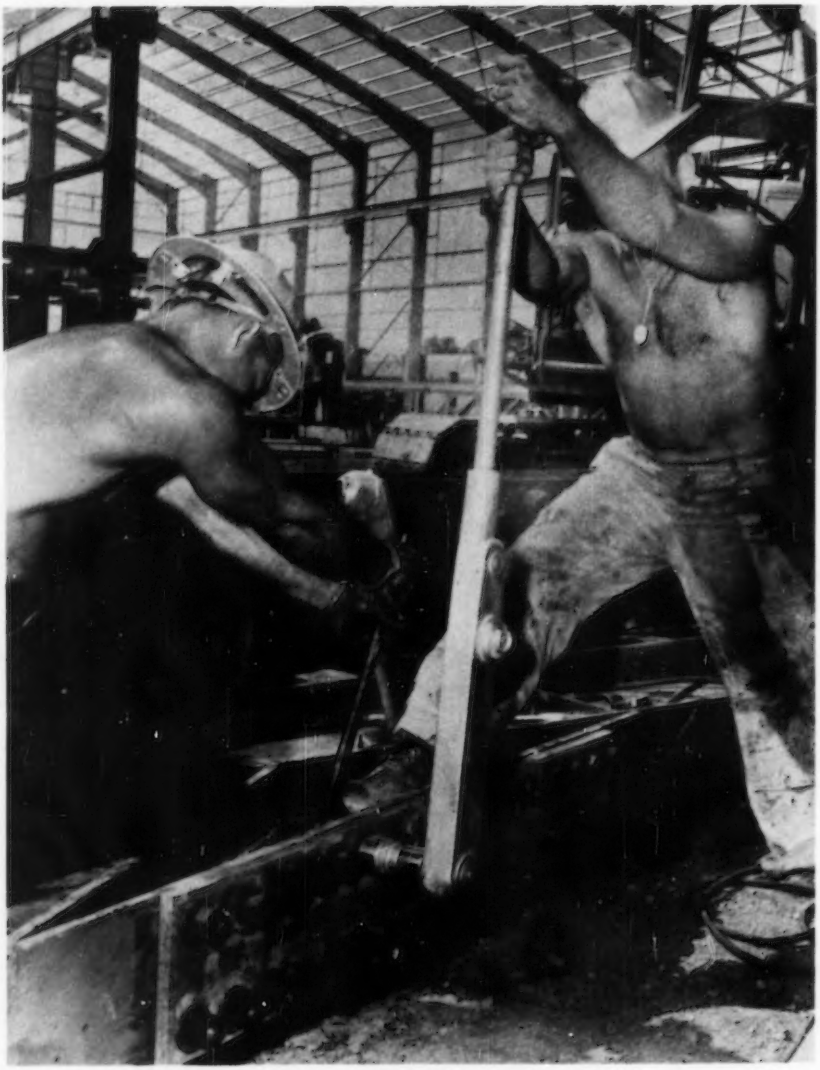


Fig. 7

Fig. 8

Fig. 9

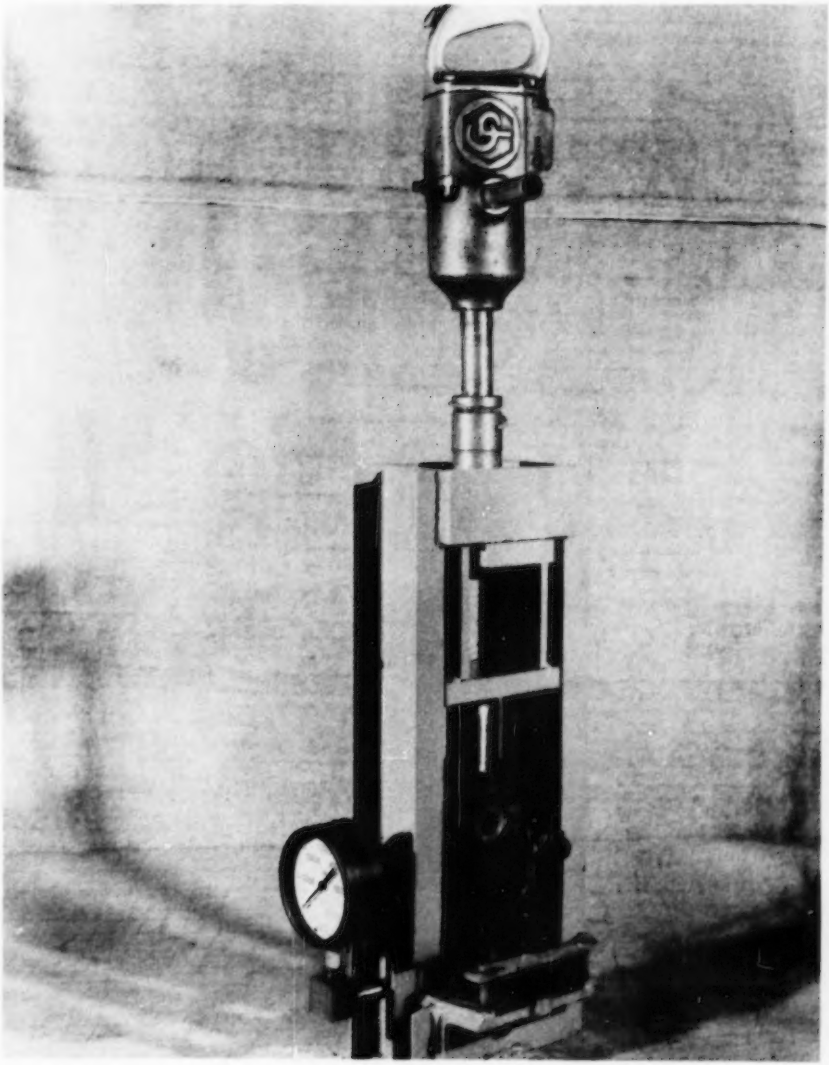


Fig. 10

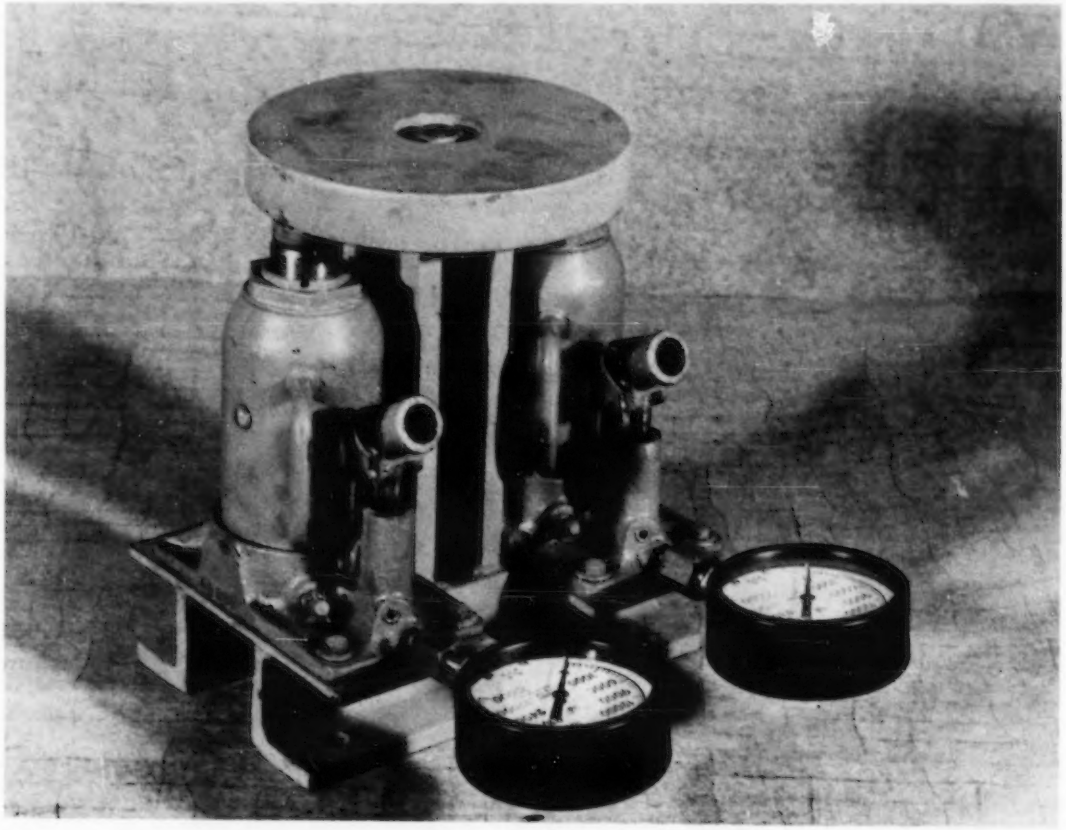
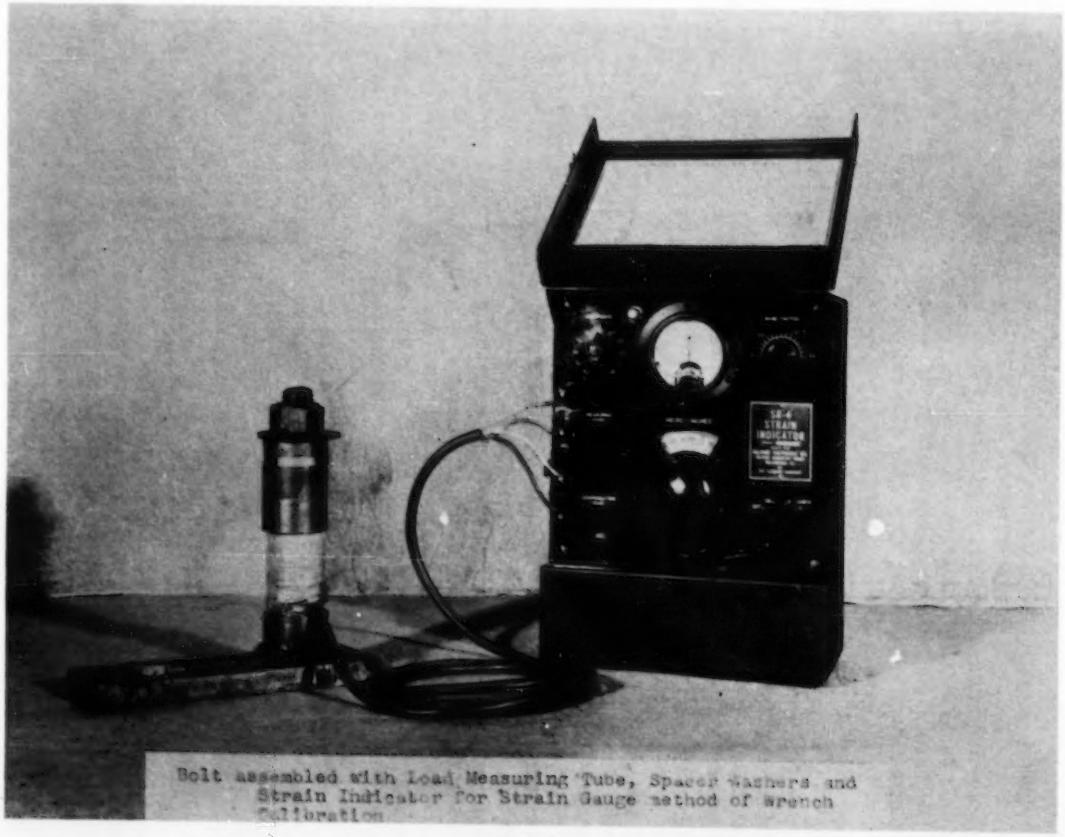


Fig. 11



Bolt assembled with Load Measuring Tube, Spacer Washers and Strain Indicator for Strain Gauge Method of Wrench Calibration

Fig. 12

The sixth and last important feature of the Council's specification covers inspection of the bolting. The specification, when first issued in 1951, recommended that between 5 and 10 percent of all bolts installed be checked by a procedure of loosening and re-tightening, and suggested that the torque required to re-tighten be equal to the approximate equivalent torque previously shown in the table Figure 6. This procedure was time consuming, and not altogether satisfactory, since at best the torque-tension relationship is apt to be somewhat erratic. With the development of the calibrating devices that have been described, all the variables which affect bolt tightness—wrench capacity, bolt size and length, and lubrication and condition of the threads—are taken into account automatically.

Four years of field experience made it increasingly evident that inspection by approval of the procedures for calibration of wrenches and installation of bolts, with follow-up observations to assure that the procedures are being followed, had the advantages of insuring even closer control of bolt tensions. With issuance of the current 1954 specification, the Council approved this method of inspection, and eliminated the requirements that torque be used as a measure of tension.

In practice, each wrench is calibrated at least once a day, using the same length of hose that will be used on the job between the wrench and the regulating valve. The inspectors check closely to make sure that the impact wrenches are correctly calibrated for the bolts that are to be installed. After the joints are "made-up," the bolts are tightened, usually starting at the center and working out towards the ends of a connection. As each bolt is tightened, a mark is placed on it to indicate that it has been tightened.

Since the first commercial installation of High Tensile bolts a little over four years ago, the use of this new type connection has grown at a phenomenal rate. It is impossible to keep track of all of the structures in which they have been used. Partial lists that were compiled indicate that they have been used in all types of buildings from hospitals, office buildings and schools, to A.E.C. plants and steel mills, and in both new construction and maintenance work on both railway and highway bridges. Special applications have also been reported, as in radio transmission towers, refinery structures, and boiler supporting frameworks.

The following group of photographs have been chosen to illustrate the wide variety of structures, each having connections suitable for their particular service conditions, that are currently being erected with High Tensile bolted field connections.

The new bottom lateral stiffening system on the great Golden Gate Bridge, designed to give the bridge additional resistance to vibrations induced by wind, is being installed with these bolts. Figure 13 shows a part of the system as installed between the main stiffening trusses. The 4700 tons of steel being placed will require some 190 tons of High Tension bolts for connections.

This photograph, Figure 14, gives a closer view of some of the joints. Due to the varying character of wind loads, these joints will be subject to stress reversals, fatigue, and vibrations.

Figure 15 is a close-up view of a connection of the new steel to the main stiffening trusses. Note the impact wrench lying on the lateral gusset plate. The workmen are checking the tightness of a bolt with a torque wrench, a practice seldom followed today.

Here is a through type plate girder railway bridge in the East, Figure 16, in which all floor beam, stringer and lateral bracing connections are made with these bolts. There have been many bridges constructed in this way

Fig. 13

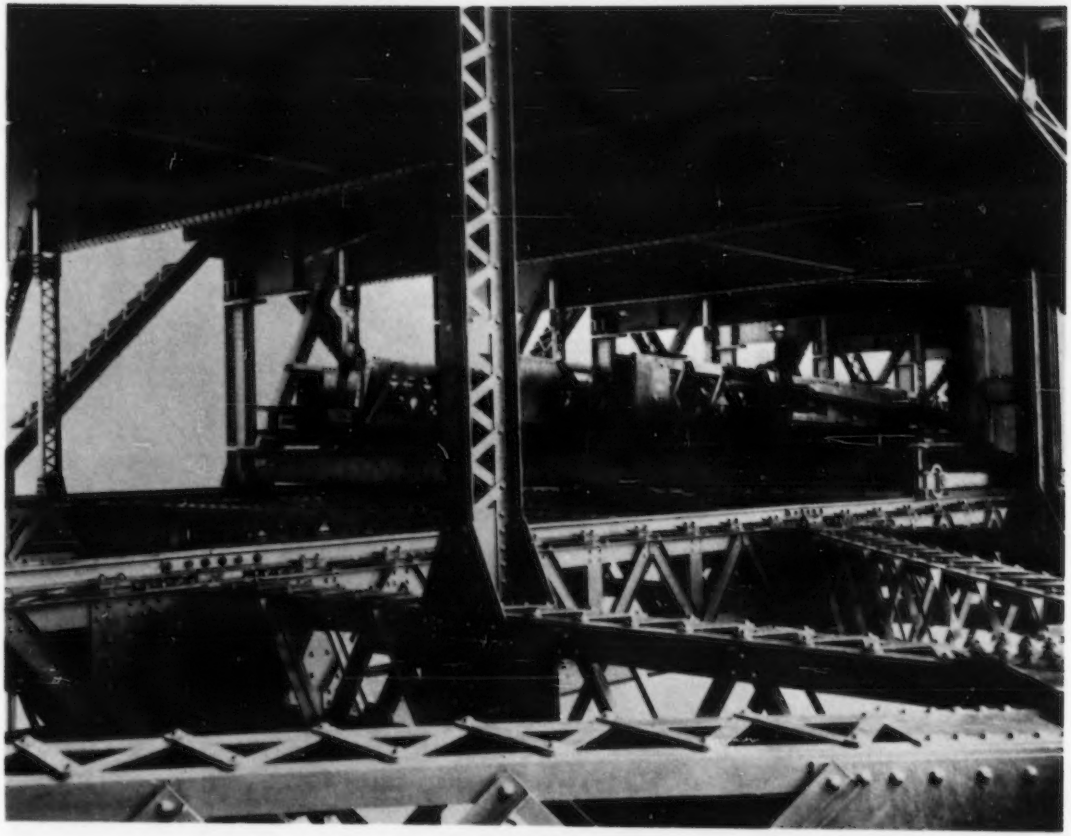


Fig. 14

Fig. 15

Fig. 16

recently in order to avoid having to move riveting equipment to a job site for a comparatively small number of connections.

The details of the floor-beam to girder connection are shown in Figure 17. Note also that the lateral bracing gusset plate and trough support beam connection details can be seen.

Figure 18 illustrates a stringer to floor-beam connection. The unpainted areas for the field bolted parts of the connection show clearly. It is of interest to note here that one angle of the connection is shop riveted to the floor beam, and that the other is bolted. Both of course are through bolted to the stringer. And Figure 19, the opposite end of this stringer, shows the end diaphragm connection and bearing detail.

Here is a close-up of bolt installationwork, Figure 20, on a crane runway knee-brace connection. Such joints are always subject to fatigue and impact. This is in a large tractor plant in the East. These heavy truss to column connections, Figure 21, are in the same job.

The framing of this automobile assembly-plant in the West, Figure 22, was erected at the rate of a half-acre a day, using high tensile bolts for field connections of all 2600 trusses involved. The building has a total of 38 bays at 45 ft. in its long dimension of 1710 ft., and 14 — 50 ft. and 1 — 90 ft. bay in its short dimension of 790 ft. (Area = 1,351,000 sq. ft. or 31 acres).

Here is a beam to column connection, Figure 23, in a rigid frame type industrial building. The moment connections of haunched girder to columns of the main frames can be seen just above the workmen's heads. This building is also located in the East.

Denver's new Mile High Center has attracted a lot of attention in construction news recently. The 23 story framework of 1700 Broadway, Figure 24, an office building, will be the dominant feature of "Mile High Center." The framework went up in just 82 days, (4500 tons). All column splices, beam to column connections, and bracing connections were made with high tensile bolts. Figure 25, pictures the bolting of a wind bracing member in this structure.

One of the most unusual frameworks ever employed in a tier building, Figure 26, is rapidly nearing completion in San Francisco. This 25 story office building for the Equitable Life Assurance Society, designed to resist earthquake forces under the rigid San Francisco code, has tapered exterior columns and spandrel beams in the lower floors. It is noteworthy too in that riveting, welding, and bolting are all being used. High strength bolting is used to splice the columns, Figure 27, and to make all beam to column connections, Figure 28, with the exception of the spandrels below the 16th floor, which are welded. Both shop riveting and welding were used in fabricating the various members.

This close-up view of the one of the exterior tapered column splices, Figure 29, illustrates well the combination of shop welding and riveting, with High Tensile field bolting.

Since the first commercial installation at Keene, N. H., four years ago, many millions of high tensile bolts have been used to erect hundreds of thousands of tons of structural steel. The Research Council is continuing its program of research, and it is easily conceivable that the results of these investigations can lead to the development of Specification provisions and installation techniques that will make the use of high tensile bolted connections even more attractive to bridge and structural engineers.

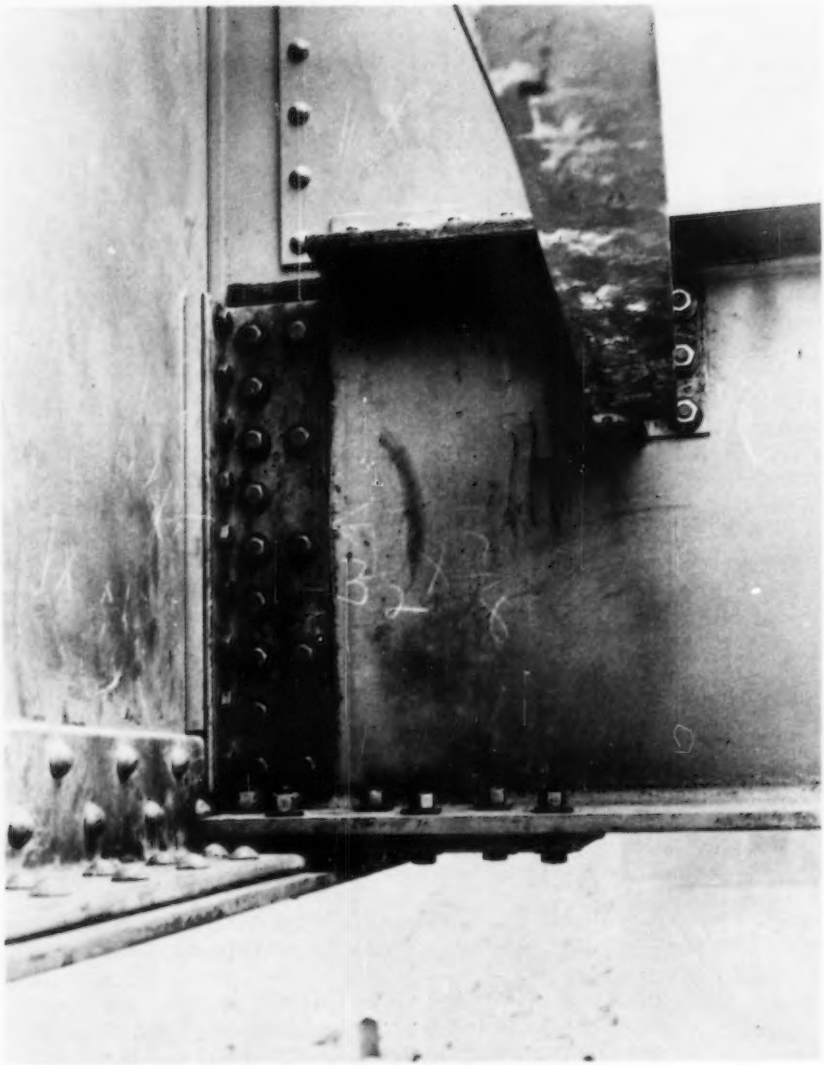


Fig. 17

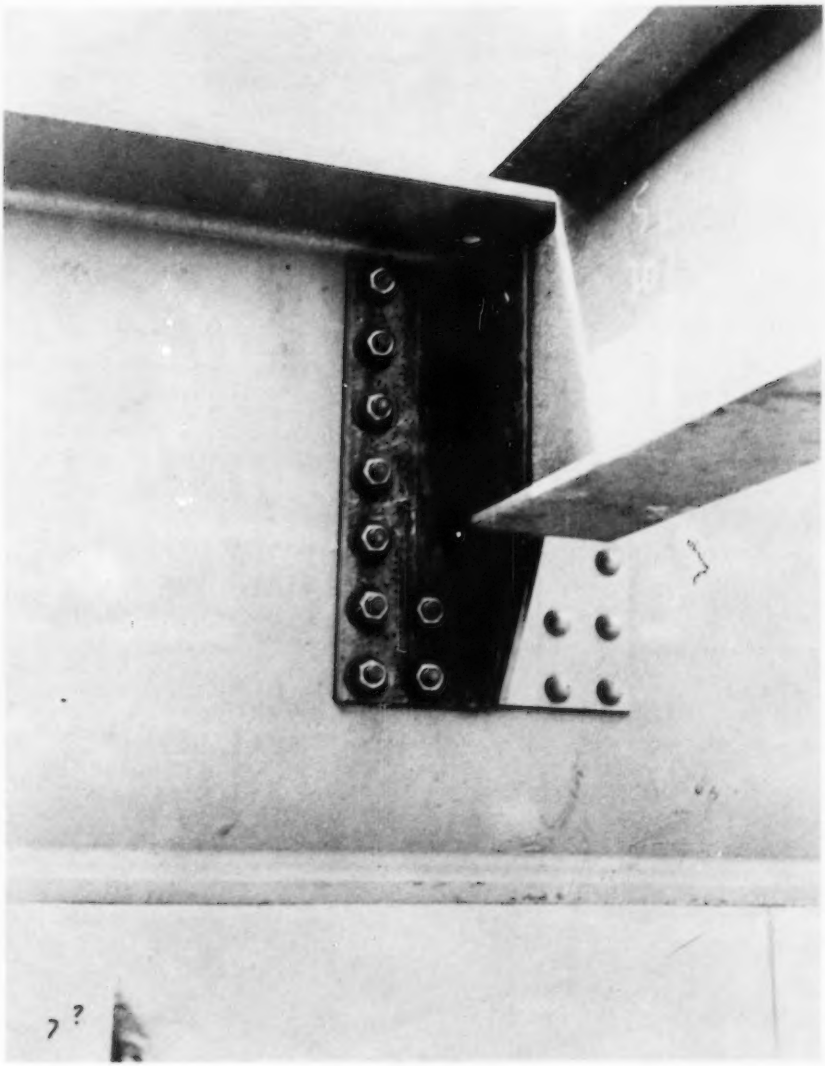


Fig. 18

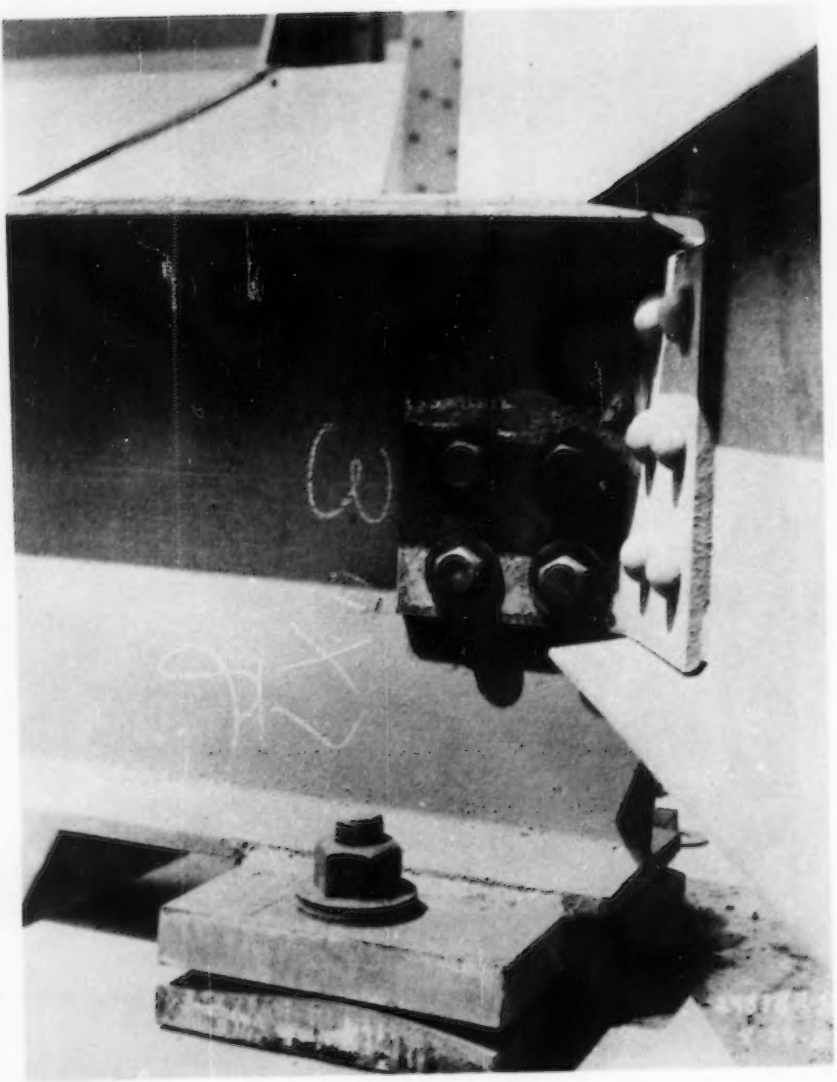


Fig. 19

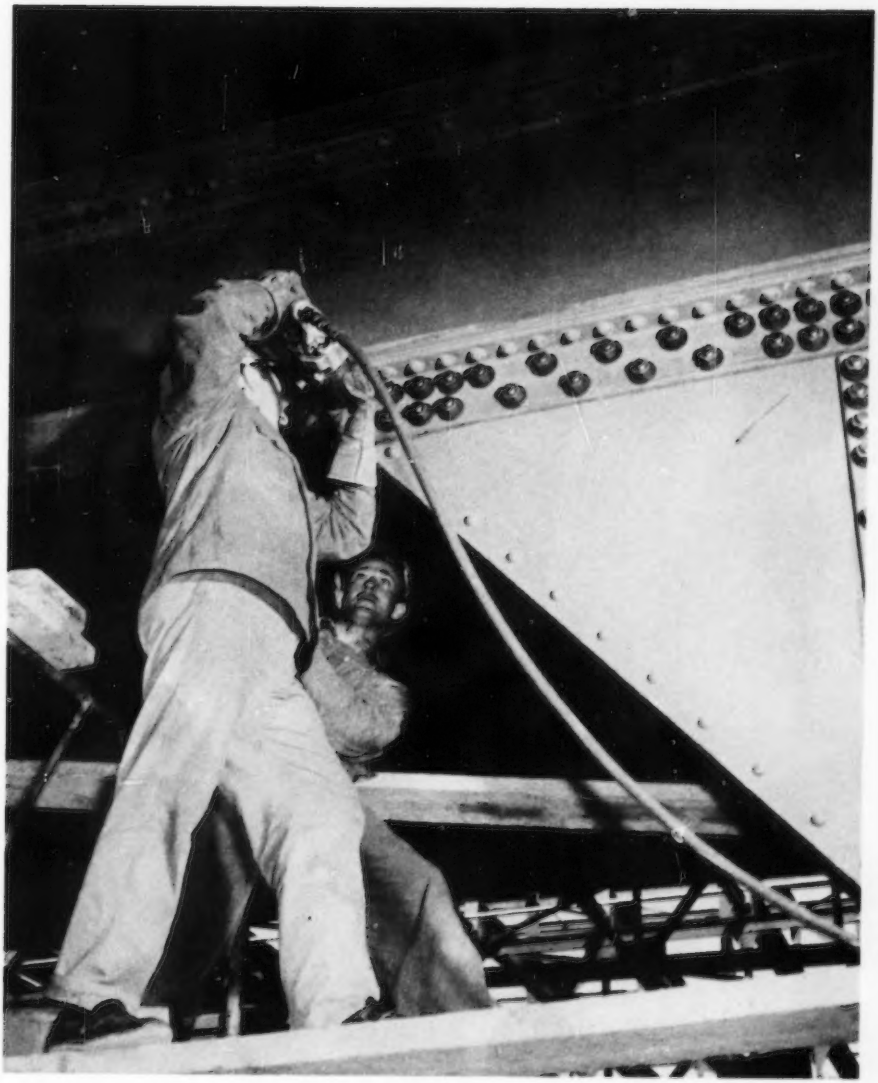


Fig. 20

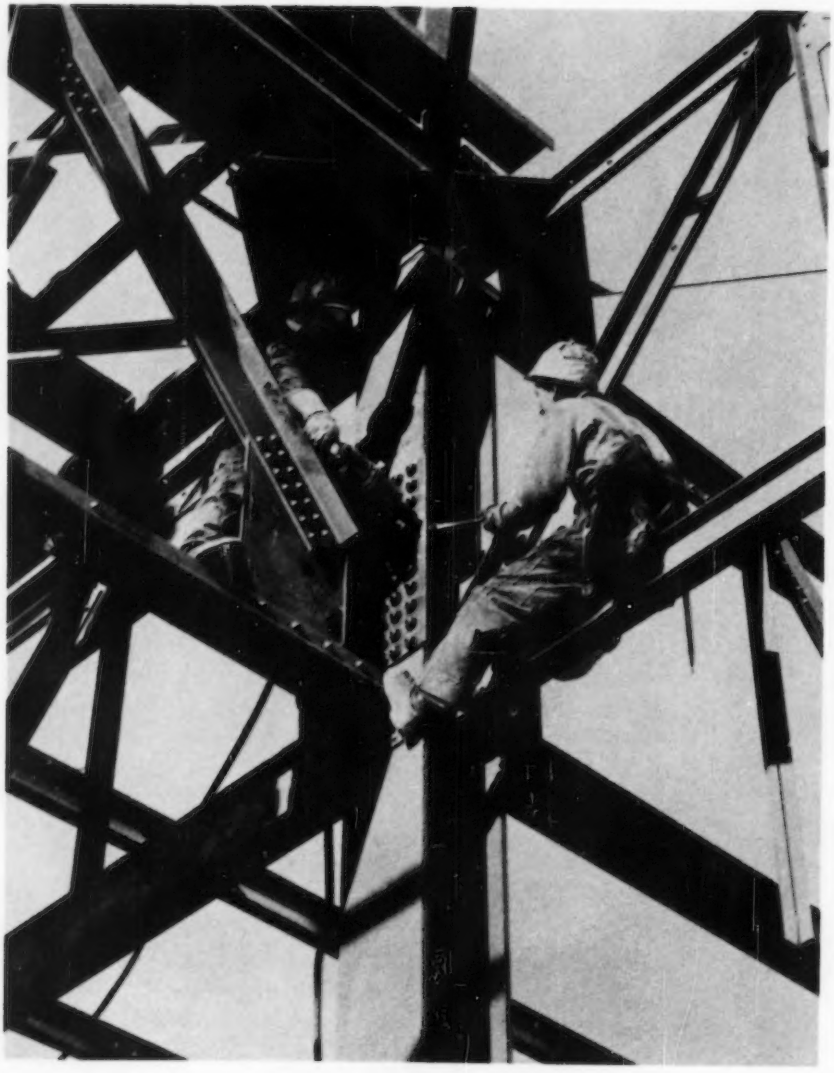


Fig. 21

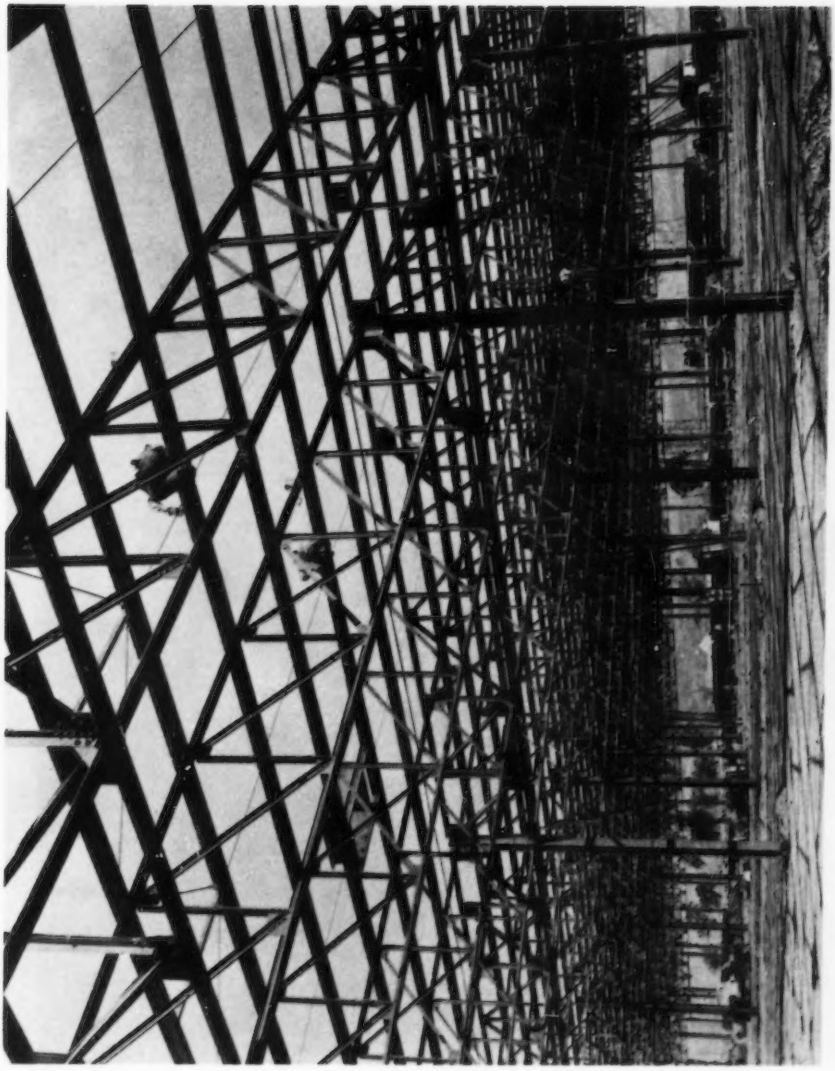


Fig. 22

Fig. 23

Fig. 24

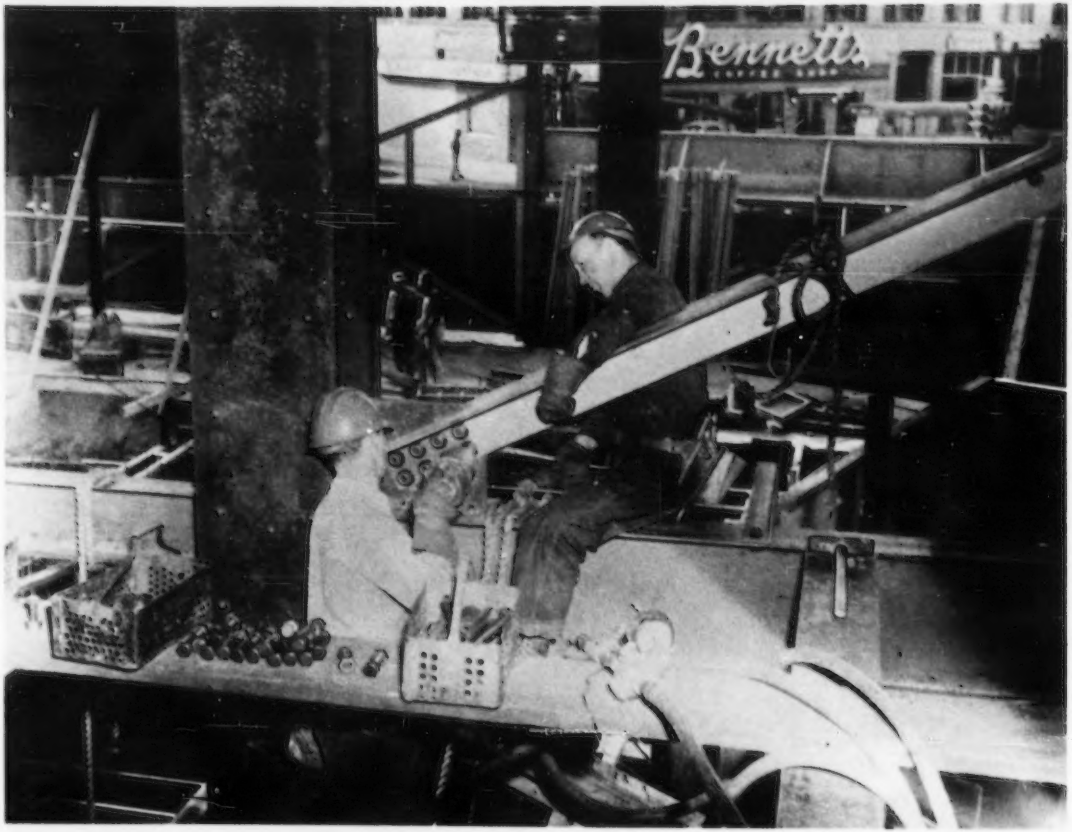


Fig. 25

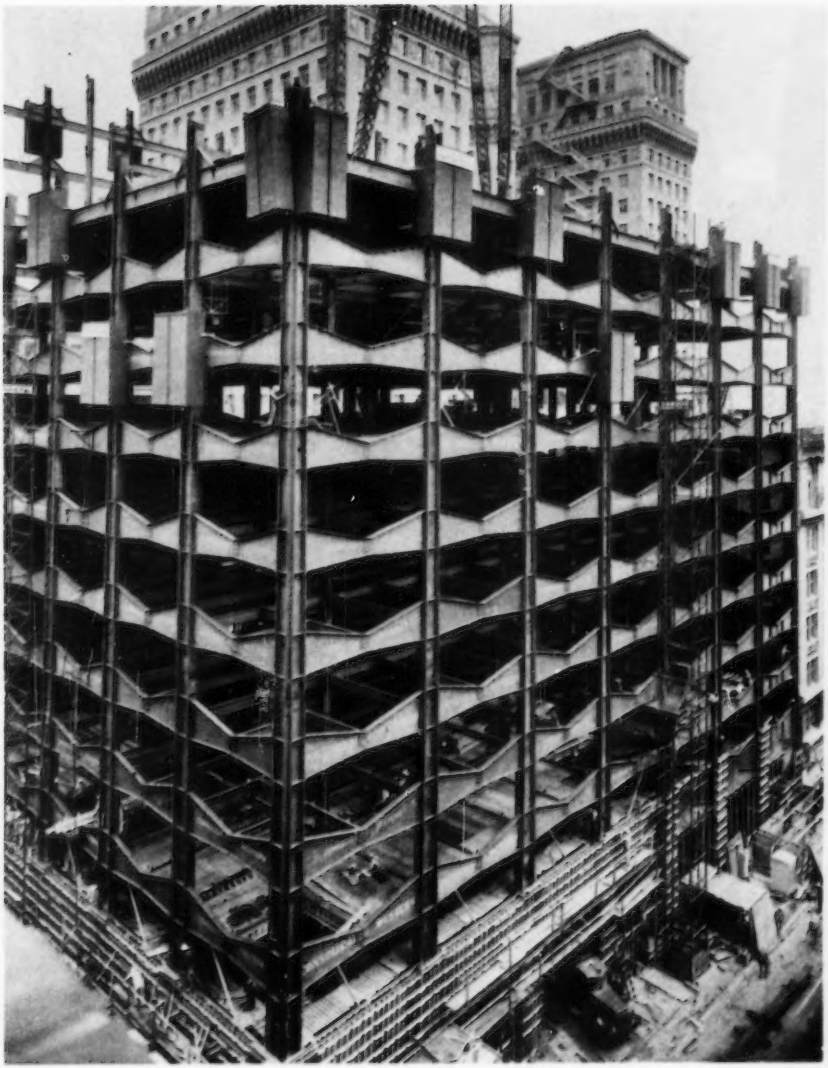


Fig. 26

Fig. 27

Fig. 28

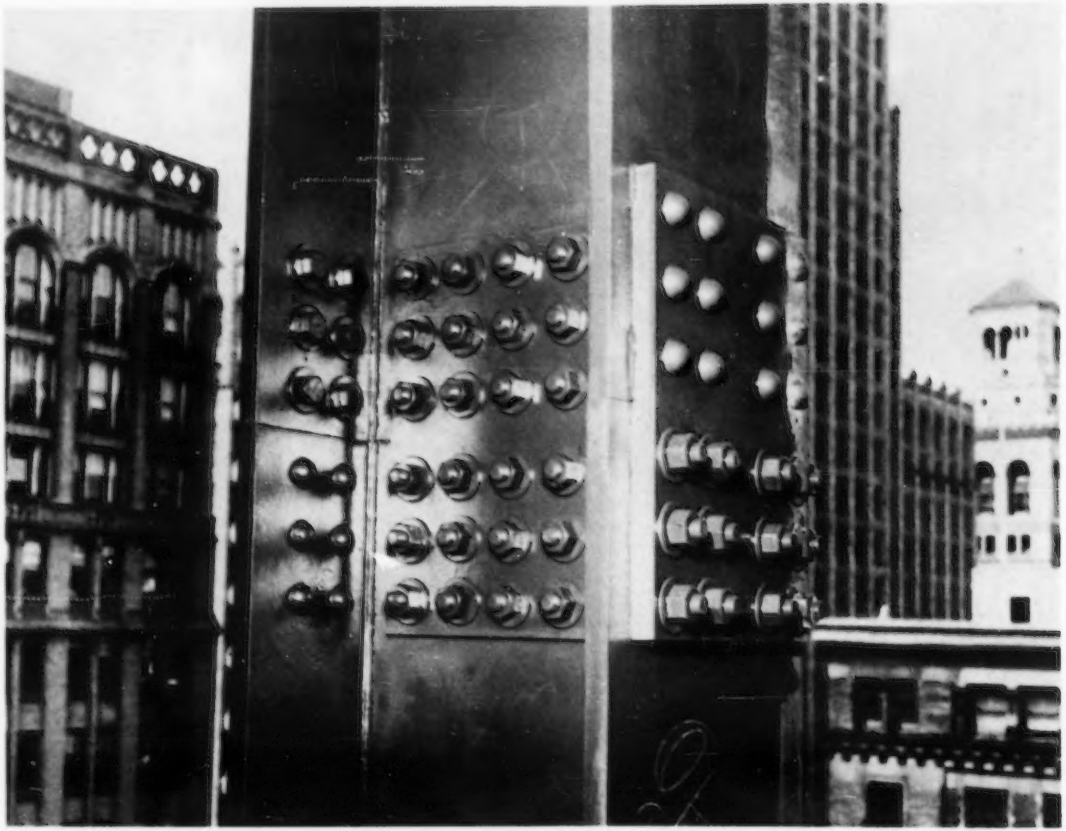


Fig. 29

PROCEEDINGS-SEPARATES

The technical-papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

VOLUME 80 (1954)

MARCH: 414(WW)[d], 415(SU)[d], 416(SM)[d], 417(SM)[d], 418(AT)[d] 419(SA)[d], 420(SA)[d], 421(AT)[d], 422(SA)[d], 423(CP)[d], 424(AT)[d], 425(SM)[d], 426(IR)[d], 427(WW)[d].

APRIL: 428(HY)[c], 429(EM)[c], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

MAY: 435(SM), 436(CP)[c], 437(HY)[c], 438(HY), 439(HY), 440(ST), 441(ST), 442(SA), 443(SA).

JUNE: 444(SM)[e], 445(SM)[e], 446(ST)[e], 447(ST)[e], 448(ST)[e], 449(ST)[e], 450(ST)[e], 451(ST)[e], 452(SA)[e], 453(SA)[e], 454(SA)[e], 455(SA)[e], 456(SM)[e].

JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

SEPTEMBER: 484(ST), 485(ST), 486(ST), 487(CP)[c], 488(ST)[c], 489(HY), 490(HY), 491(HY)[c], 492(SA), 493(SA), 494(SA), 495(SA), 496(SA), 497(SA), 498(SA), 499(HW), 500(HW), 501(HW)[c], 502(WW), 503(WW), 504(WW)[c], 505(CO), 506(CO)[c], 507(CP), 508(CP), 509(CP), 510(CP), 511(CP).

OCTOBER: 512(SM), 513(SM), 514(SM), 515(SM), 516(SM), 517(PO), 518(SM)[c], 519(IR), 520(IR), 521(IR), 522(IR)[c], 523(AT)[c], 524(SU), 525(SU)[c], 526(EM), 527(EM), 528(EM), 529(EM), 530(EM)[c], 531(EM), 532(EM)[c], 533(PO).

NOVEMBER: 534(HY), 535(HY), 536(HY), 537(HY), 538(HY)[c], 539(ST), 540(ST), 541(ST), 542(ST), 543(ST), 544(ST), 545(SA), 546(SA), 547(SA), 548(SM), 549(SM), 550(SM), 551(SM), 552(SA), 553(SM)[c], 554(SA), 555(SA), 556(SA), 557(SA).

DECEMBER: 558(ST), 559(ST), 560(ST), 561(ST), 562(ST), 563(ST)[c], 564(HY), 565(HY), 566(HY), 567(HY), 568(HY)[c], 569(SM), 570(SM), 571(SM), 572(SM)[c], 573(SM)[c], 574(SU), 575(SU), 576(SU), 577(SU), 578(HY), 579(ST), 580(SU), 581(SU), 582(Index).

VOLUME 81 (1955)

JANUARY: 583(ST), 584(ST), 585(ST), 586(ST), 587(ST), 588(ST), 589(ST)[c], 590(SA), 591(SA), 592(SA), 593(SA), 594(SA), 595(SA)[c], 596(HW), 597(HW), 598(HW)[c], 599(CP), 600(CP), 601(CP), 602(CP), 603(CP), 604(EM), 605(EM), 606(EM)[c], 607(EM).

FEBRUARY: 608(WW), 609(WW), 610(WW), 611(WW), 612(WW), 613(WW), 614(WW), 615(WW), 616(WW), 617(IR), 618(IR), 619(IR), 620(IR), 621(IR)[c], 622(IR), 623(IR), 624(HY)[c], 625(HY), 626(HY), 627(HY), 628(HY), 629(HY), 630(HY), 631(HY), 632(CO), 633(CO).

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA)[c], 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1955

PRESIDENT

WILLIAM ROY GLIDDEN

VICE-PRESIDENTS

Term expires October, 1955:
ENOCH R. NEEDLES
MASON G. LOCKWOOD

Term expires October, 1956:
FRANK L. WEAVER
LOUIS R. HOWSON

DIRECTORS

Term expires October, 1955:
CHARLES B. MOLINEAUX
MERCEL J. SHELTON
A. A. K. BOOTH
CARL G. PAULSEN
LLOYD D. KNAPP
GLENN W. HOLCOMB
FRANCIS M. DAWSON

Term expires October, 1956:
WILLIAM S. LaLONDE, JR.
OLIVER W. HARTWELL
THOMAS C. SHEDD
SAMUEL B. MORRIS
ERNEST W. CARLTON
RAYMOND F. DAWSON

Term expires October, 1957:
JEWELL M. GARRELTS
FREDERICK H. PAULSON
GEORGE S. RICHARDSON
DON M. CORBETT
GRAHAM P. WILLOUGHBY
LAWRENCE A. ELSENER

PAST-PRESIDENTS

Members of the Board

WALTER L. HUBER
DANIEL V. TERRELL

EXECUTIVE SECRETARY

WILLIAM N. CAREY

ASSOCIATE SECRETARY

WILLIAM H. WISELY

TREASURER

CHARLES E. TROUT

ASSISTANT SECRETARY

E. L. CHANDLER

ASSISTANT TREASURER

CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Assoc. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

SAMUEL B. MORRIS, *Chairman*

JEWELL M. GARRELTS, *Vice-Chairman*

GLENN W. HOLCOMB
ERNEST W. CARLTON
OLIVER W. HARTWELL
DON M. CORBETT